Prepared for The City of San Jose

STABILITY ANALYSIS
FOR LANDSLIDE STABILIZATION
ALUM ROCK AVENUE,
SAN JOSE, CALIFORNIA

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FLAC-FINITE DIFFERENCE
& LIMIT STABILIZATION EQUILIBRIUM STABILITY ANALYSIS
FOR LANDSLIDE STABILIZATION AT THE
ALUM ROCK PARK ENTRANCE, SAN JOSE, CALIFORNIA

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1.0 INTRODUCTION

This report presents the results of our geotechnical analysis and geologic field investigation for the Alum Rock Avenue Landslide in San Jose, California. Copies of the geotechnical investigation report, boring logs and inclinometer data used in our analysis were provided by the City of San Jose, Department of Public Works. A vicinity map showing the location of the project site is presented below:

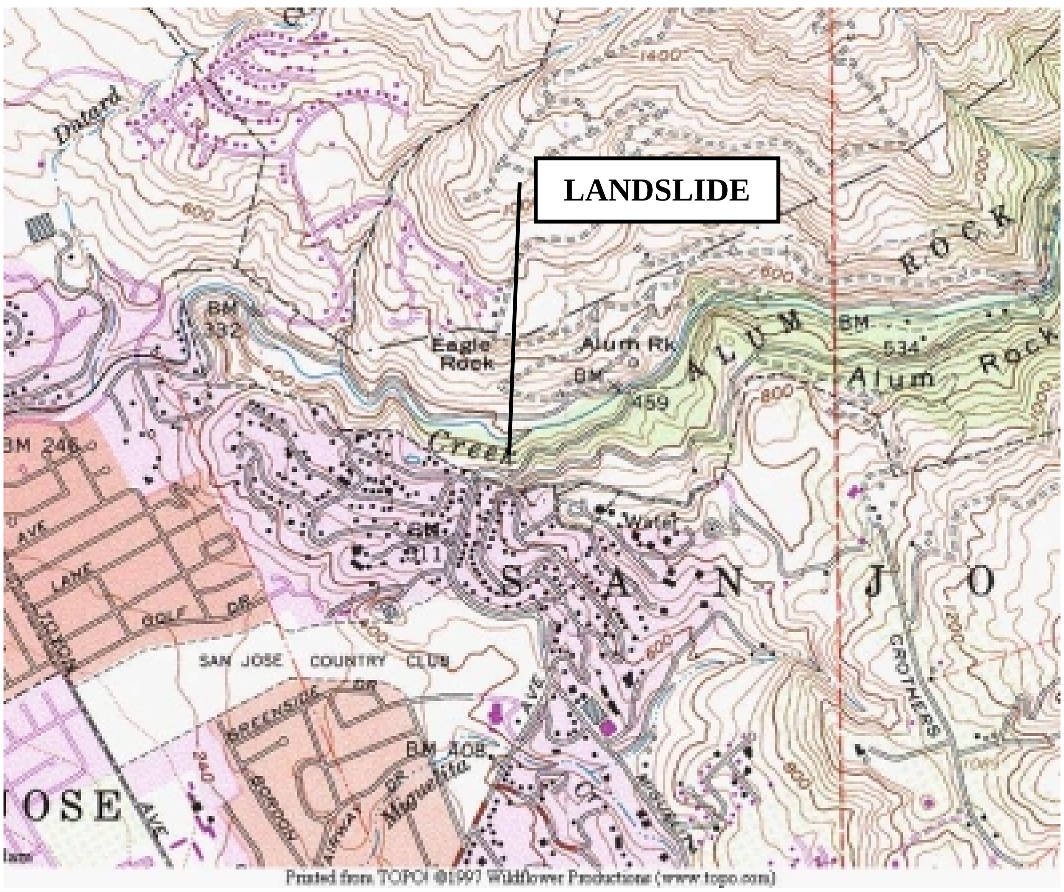

Figure 1 – Site Location

The available geotechnical reports used for our analysis include:

- "Phase I Geotechnical Investigation, Evaluation for Emergency Protective Measures, Crothers Road Area Landslide, Santa Clara, California", prepared by Earth Tech, January, 1999;
- "Horizontal Drains Installation Technical Specifications (90% submittal)", prepared by Earth tech, July, 1999; and,
- "Additional Inclinometers Installation and Monitoring", prepared by Earth Tech, December 1999.

1.1 Background

The Alum Rock Avenue Landslide is located on a north-facing slope of the Alum Rock Park in San Jose. Active, documented landslides have occurred on this site since the heavy rains during the winter of 1997 through 1998. Four roads, Alum Rock Avenue, Crothers Road, Highland Drive and a maintenance road, cross the zone where active sliding has occurred. Currently, the slide extends to Highland Drive near the top of the slope where a 4 to 5 foot high headscarp has developed in the roadway.

1.2 Scope

The scope of our services includes the following:

- Conduct a review of all available previous relevant geotechnical investigations, aerial photographs and slide maps for the area;
- Conduct a site reconnaissance to visually observe the features of the slide area;
- Perform slope stability and other engineering analysis using the available field and laboratory findings from engineering studies by others.
- Construction of a numeric model of the slide area using XSTABL and FLAC computer programs;
- An engineering evaluation of alternative slope stabilization methods;
- Meetings with the city, as required to present findings of our analyses;

- Preparation of a cost estimate for the most feasible alternative for permanent reconstruction of Alum Rock Avenue at the park entrance; and,
- Submit a final detailed report.

2.0 GEOLOGY

2.1 Local and Site Geology

The geologic map of the Calaveras Reservoir quadrangle (Dibblee, 1973), a portion of which is included as Plate 2A, shows the landslide to be mostly within the Berryessa formation. The landslide was not mapped by Dibblee; however, we have drawn the approximate limits on this map to better illustrate it relationship to the surrounding geology. The Berryessa formation is part of the Cretaceous age Great Valley sequence of deep marine sedimentary rocks, generally composed of shale with some interbedded sandstone. These rocks are generally highly weathered, and closely to intensely fractured in and around the landslide area. The geologic map indicates that the more resistant Oakland conglomerate formation crops-out immediately to the west-southwest of the landslide side scarps as well as approximately 900 feet to the east-northeast. Dibblee (1973) shows that the general bedding attitude of the Berryessa and Oakland Conglomerate rocks in the vicinity strikes northwesterly and dips moderately (50 to 65 degrees) to the northeast.

The road cuts on Alum Rock Avenue, the maintenance road and Crothers Road expose beds of the Berryessa formation shale and sandstones. Measurements taken on the road cuts east of the slide indicate that the bedding strikes between 170 and 250 degrees (southwest) with dips between 20 and 32 degrees to the southeast. Measurements of a joint set recorded at the same bedrock exposures strike between 30 and 85 degrees (northeast) with dips between 22 and 35 degrees northwest (see Plate 2C). These bedding and joint orientation, in and of themselves, were insufficient to draw any conclusion regarding the entire slide. No outcrops of the Oakland Conglomerate were observed within the immediate vicinity of the project site.

Shallow colluvial deposits generally conceal the bedrock on natural slopes and thick colluvial deposits exist within most of the larger swale areas illustrated on the Photointerpretation of Landslides Map, Plate 2C, from Nilsen and Brabb (1972). Colluvial material consists of poorly stratified and weak deposits of silt and clay with highly weathered sandstone and shale rock fragments, and variable content of organic material. These deposits have accumulated by the process of gradual downslope migration of soil constituents, by the action of gravity and rainfall run-off.

Holocene alluvium lies within the relatively flat area around Penitencia Creek below the project landslide. This material is composed of stratified and unconsolidated gravel, sand, silt and clay mixtures. Migration of the Penitencia Creek channel resulted in episodic deposition and erosion of alluvium within the bottom of the canyon. Currently, the creek channel is located at the base of the landslide on the west side of the canyon and is incised below the adjacent alluvial plain several feet.

2.2 Local Faulting

The site is located near the southern limit of the Hayward fault zone where it merges with the Calaveras fault. In this area the Hayward fault zone is referred to as the Hayward fault – southeast extension and consists of numerous mapped short fault segments over a zone more than a mile wide. Older maps of this area (Dibblee, 1973 and Rogers and Williams, 1974) show several fault traces in the vicinity including the Hayward fault mapped about 4000 feet northeast of the site (not shown on Plate 2b), the Crosley fault mapped a few hundred feet north of the site (shown just across Penitencia Creek on Plate 2b) and the Evergreen fault mapped about 2000 feet southeast of the site (shown as generally concealed beneath alluvium on Plate 2b). These faults are generally considered to have been active during the Late Quaternary, but not active during the Holocene (Jennings, 1994 and Bortugno, 1991). The active Calaveras fault zone lies approximately 2 miles northeast of the landslide.

In personal communication with Sandy Figures of Norfleet Consultants, who studied the Alum Rock area geology in detail, we have learned that many of the faults may be very large landslide slip surfaces rather than earthquake faults. Norfleet Consultants believe that the Hayward fault does not extend south of Milpitas.

2.3 Landslide

The United States Geological Survey (USGS) has prepared landslide maps for most areas of the San Francisco Bay Area during the 1970s. Nilsen (1972), a portion of which is illustrated on Plate 2C, covers the Alum Rock Park area. Several large landslide features are shown in the vicinity; infact, this particular landslide was mapped on Nilsen (1972). We have drawn-in the approximate limits of the landslide on Plate 1A.

The Nilsen landslide maps prepared by the USGS are based on interpretation of aerial photographs. We have also examined aerial photographs: AV-1277, Scale 1:12,000 (11-04-76); AV 3324, Scale 1:12,000 (06-28-88); AV-5200, Scale 1:12,000 (07-31-96). A bowl shaped feature, indicating an old landslide is visible on the photographs. However, clear indications of recent landslide movement over this time period were not evident. This may be a result of the vegetation cover of numerous mature trees and dense brush. The limits of the currently active landslide are entirely within this larger, landslide feature.

Plate 1, West side of toe of landslide

The landslide deposits are composed of weathered sandstone and shale bedrock, colluvial material and artificial fill. The colluvial deposits within the landslide can be described as poorly stratified to unstratified, weak deposits of silt, sand and clay with weathered sandstone and shale rock fragments and organic material. The artificial fill is primarily related to embankments, for the roads crossing the landslide and appear to be derived from excavation made to create the roads.

Geologic field mapping and review of previous geologic investigations (Earth Tech, 1999) indicate that the currently active landslide consists of numerous smaller failures within a larger composite landslide. The landslide is approximately 560 feet (horizontal distance) in length from the main headscarp at Highland Drive to the toe at Penitencia Creek. It is approximately 280 feet wide between the main sidescarps at the widest point. The approximate limits of the composite landslide are shown on Plate 1A. Within the landslide, the ground surface has gradients ranging between approximately 3:1 and 1:1½ (horizontal to vertical).

The Geologic Map Plate 1A is based primarily on the previous mapping by Earth Tech. The limits of the recently active landslide are indicated by numerous open cracks, some with vertical displacement of several inches. These cracks form an arcuate shape in the upper reaches (headscarp area) which extend beyond highland Drive onto the lot at 16196 Highland Drive. Additional minor cracking with small displacements (less than one inch) within the ground surface, pavement surface, a retaining wall along the west side of Highland Drive and exterior structural improvements at 16196 Highland Drive were also observed beyond the mapped landslide headscarp. These are recorded on Plate 1A. The lateral scarps are evident by discontinuous cracks showing horizontal and vertical displacements as well as the side scarps of the smaller landslides within, the main slide mass.

Our reconnaissance identified new cracks and slumps within the landslide limits, not previously mapped, on and through the asphalt concrete sections of Alum Rock Avenue, the maintenance road and Crothers Road. These new features are shown on Plate 1a. Some cracks and slumps that had previously been repaired or overlayed had re-opened. New cracks and slumps on the roads were observed outside the previously mapped slide limits. New cracks were also found above the previously mapped headscarp on the west side of Highland Drive as well as in the driveway, within the front retaining wall, and at the rear porch area of the residence located at 16196 Highland Drive.

The eastern section of the landslide toe, adjacent to Penitencia Creek, is undergoing active erosion and slumping. At this location, recent activity is primarily associated with movement of the active sub-slide at the northeast portion of the larger slide. Approximately 60 feet west of the eastern sidescarp, some eight feet above the Creek level at the toe of the landslide, a basal slide plane was clearly evident, with relatively undisturbed and well bedded shale overlain by approximately 2 inches of a fat clay "gouge zone" (see Plate 1B). The bedding and joint fracture orientations observed in the underlying bedrock are consistent with the in-place bedrock exposures observed within the road cuts east of the landslide (See Photo No. 2, Section 4.2). The gouge zone contains numerous polished planes with striations oriented parallel to the long axis of the landslide. This gouge zone material is typical of landslide basal failure surfaces. The gouge zone is overlain by weathered and crushed shale material exhibiting severely disrupted structure. The gouge zone appears to extend below the flow line of Penitencia Creek in the central portion of the landslide toe area.

The borings drilled within the landslide by Earth Tech encountered 6 to 15 feet of colluvium and fill material overlying severely weathered bedrock (predominantly shale with some sandstone) to depth of 30 to 35 feet below the ground surface. To depths of 30 to nearly 60 feet, the bedrock material is described as severely to moderately weathered with numerous crushed and sheared zones, particularly within the shale. Below depths of 50 to 60 feet, the bedrock material is described as relatively fresh with well-preserved bedrock structure.

3.0 CONCLUSIONS

3.1 Landslide Configuration and Movement

The Alum Rock Avenue landslide is an ancient landslide, which has apparently been reactivated by the extremely wet winter of 1997-1998. It has experienced some additional movements as a result of the 1998-1999 and 1999-2000 winters as well. Our interpretation of the landslide limits is shown on Plate 1A. Surface cracks were observed upslope beyond the limits previously mapped by Earth Tech. This is based on our observation of new cracks and displacement features on Highland Drive and the lot at 16196 Highland Drive. The slope indicator data does not indicate that significant movement has occurred at depth above the headscrap; however, the numerous surface cracks indicate that near surface creep of the area above the headscarp is occurring, likely as a result of the loss of lateral support at the landslide headscarp. Continued movements of the landslide should be expected in the years to come, particularly during the winter and spring.

Numerous cracks and minor slump features within and adjacent to Alum Rock Avenue, the maintenance road and Croathers road are shown on Plate 1A to be outside of the landslide limits. These features are interpreted to be related to shallow subsurface creep of the roadway embankments or localized instability of the cut slopes rather than deep-seated landslide movement below. These features are well developed on Alum Rock Avenue and the maintenance road, west of the landslide indicating that the roadway embankments have been weakened.

The landslide is interpreted to be a translational composite landslide involving both weathered bedrock and overlying soil deposits. The landslide exhibits numerous active sub-slides within a larger composite landslide. Additionally, a downslope dipping joint set was observed within bedrock exposures along the east side of the landslide, suggesting that jointing may have provided an adverse planar weakness upon which landslide movement has occurred. A well developed gouge zone has been identified at the toe of the landslide and within some borings at the interface between the fresh and weathered bedrock. Inclinometer data has shown a slide plane to be located at the upper surface of the relatively fresh bedrock in the gouge zone.

Section B-B’, Plate 1b, shows our generalized interpretation of the subsurface profile through the landslide based on the boring data by Earth Tech, our reconnaissance, and inclinometer monitoring. The subsurface materials are divided into colluvium and fill, severely weathered bedrock, severely to moderately weathered bedrock, and relatively fresh bedrock. Based on the observed gouge zone, its relationship to the bedrock weathering zones, structure and condition of the bedrock observed beneath the gouge zone, and displacements observed within the inclinometers, we have interpreted the lower zone of movement or basal failure surface to be generally at the upper surface of the rock material which is described as relatively fresh with undisturbed rock structure.

4.0 PREVIOUS WORK AND FIELD OBSERVATIONS

4.1 Data from Previous Investigations

The city of San Jose provided our office with reports prepared by Earth Tech, Inc. and Kane Geotech for our use in analyzing the Alum Rock Avenue Landslide. Since June of 1998, Earth Tech has installed five inclinometers and four piezometers in the landslide. Kane Geotech has monitored the inclinometers and has grouted Time Domain Reflectometry Cables in three of the borings. For reasons of accessibility all of the borings, slope indicators, and piezometers are located in the upper half of the slide zone.

We have reviewed the work prepared by Earth Tech and Kane Geotech and have summarized the data they recorded in the following table:

Table 1: Inclinometer Data

Inclinometer Number	Location	Depth of Installation (ft.)	Displacement at Depth	Dates Readings Taken	Remarks
1	Alum Rock Avenue	60	1.4" @ 35' 0.4" @ 54'	6/98 – 3/99	Casing Deformed, TDR cable Grouted into inclinometer casing
2	Crothers Road	68	0.25" @ 50'	6/98 – 1/99	Casing Deformed, TDR cable Grouted into inclinometer casing
3	Highland Drive	70	0.1" @ 35'	6/98 - 3/99	TDR cable grouted into inclinometer casing
4	Crothers Road	98.5	1.3" @ 34' 0.1" @ 74'	5/99 - 7/99	Casing Deformed, Unreadable. Installed 18-feet west of I-2
5	Alum Rock Avenue	99.8	1.75" @ 52'	5/99 - 11/99	Installed 21-feet west of I-1

All inclinometer monitoring performed by Earth Tech/Kane Geotech

The location of the five inclinometers and the depths of recorded movement for each have been plotted on the cross-section on Plate 1B. We understand that inclinometers 1, 2, and 3 were installed at about the same time, but that I-2 and I-1 deformed and became unreadable in January, and March respectively of 1999. Time domain reflectometer cables were grouted into the three inclinometer casings in May of 1999 and at the same time two additional inclinometers were installed. The new inclinometers were installed to approximately 100-feet deep each, and were located near the original inclinometers number 1, and 2. Since May, Kane Geotech has performed periodic readings of the inclinometers and the TDR cables and has provided a series of reports detailing the movement of the slide.

As can be seen from the table above, slide movement has been measured as deep as 74-feet below the surface of the slope on inclinometer number 4. The primary movement of the slide mass was observed between 30, and 50-feet from the ground surface in inclinometers 1, 4, and 5. The movement at 74-feet was not detected until 2 months after the inclinometer installation. At that time, the cumulative movement at 34-feet was 1.3-inches. Because of these reasons, as well as the fact that no movement has been recorded at the 74 foot depth on I-5, which is downslope from I-4 it is our opinion that there is not a significant slide deeper than 50-feet below the ground surface at this site. In order to confirm this assumption, we recommend that continued monitoring using inclinometers be carried out near the location of the initial three inclinometers.

The TDR cables have not yet recorded movement but it is believed that this is due to the inclinometer casings shielding the cables from slope movement. The TDR’s are expected to yield data once the casings rupture and the cables sense the movement of the slope.

4.1.1 Groundwater Measurements

Groundwater at the site has been monitored in four piezometers that were installed during the initial investigation in June 1998. Piezometers 1, and 2 are located on Alum Rock Avenue near the east and west edges of the slide respectively. Piezometers 3, and 4 are located along Crothers Road on the east and west edges of the slide respectively. The following table has been recreated from the Earth Tech report dated December 23, 1999 with the exception of the final reading, which was performed by Kleinfelder personnel.

Table 2: Groundwater Elevations

Boring/Piezometer No.	P-1	P-2	P-3	P-4
Ground Surface Elevation	248.3	249.7	308.9	280
Top of Pipe Elevation	247.8	249.1	308.7	279.7
Total Piezometer Depth (ft.)	40.3	40.3	40	38
6/30/98 Depth to Water/ Elevation W.L. Surf.	20.0/227.8	NA	29.2/279.5	NA
7/1/98 Depth to Water/ Elevation W.L. Surf.	22.3/225.3	NA	29.4/279.3	29.8/249.9
7/28/98 Depth to Water/ Elevation W.L. Surf.	21.8/226.0	Dry	30.8/277.9	33.3/246.4
8/10/98 Depth to Water/ Elevation W.L. Surf.	22.5/225.3	Dry	31.4/277.3	34.3/245.4
5/12/99 Depth to Water/ Elevation W.L. Surf.	22.3/225.5	19.6*	30.7/278.0	32.9/246.8
6/18/99 Depth to Water/ Elevation W.L. Surf.	23.2/224.4	19.6*	31.2/277.5	26.4/253.3
11/15/99 Depth to Water/ Elevation W.L. Surf.	25.4/222.4	4.5*	34.8/273.9	9.3/270.4
2/11/00 Depth to Water/ Elevation W.L. Surf.**	27.3/220.5	19.3/229.8	31.5/277.2	33/246.7

Notes:

1. Elevations are based on assumed base elevation and not on Mean Sea Level
2. * Constricted
3. Data from Earth Tech report dated December, 1999.
4. ** Reading by Kleinfelder Personnel

The depth to groundwater measured in the piezometers has remained fairly constant since the piezometers were first installed, See Table 2. Piezometer 4 shows a 17-foot rise in elevation but according to the Earth Tech report surface water was observed running into the piezometer so the data is considered unreliable.

Plate 25 shows a chart of monthly rainfall data recorded at Mount Hamilton near the site where the slide occurred. As can be seen in the chart, the heaviest rainfall at the site had occurred prior to the installation of the piezometers at the site. The piezometers have not been monitored in any of the winter or early spring months during which the heaviest rainfall has occurred. For these reasons, we allowed the groundwater elevation to vary in our analysis so that we might observe the behavior of the slope under the worst case situation.

4.2 Field Observations

During our field observations, a clay seam was located at the toe of the slope on the east side of the slide. The seam overlies a weathered, but competent sandstone bedrock and is overlain by landslide deposits. It is approximately 2 inches thick and is a highly plastic soil with a cohesive strength of 550, to 650 pounds per square foot as measured with a Torvane. The seam dips to the west and is projected to extend below the creek near the centerline of the slide. It was found in three locations from east to west across the toe of the slope. The adjacent photograph shows the clay seam over sandstone. The seam is also located on our cross sections shown on Plates 1B, 22, 23, and 24. Based on our field observations, and on the boring logs provided for this project, we assumed that the failure plane was located in the clay gouge material located at various elevations in the highly fractured rock slope. The clay seam was used to locate the bottom of the slide plane for the stability analysis and establish the shear strength along the slide plane.

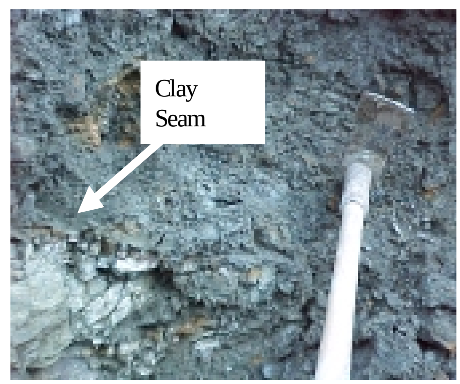

Photo No. 2

Clay Seam Observed at Toe of Slope

4.3 Installation of Inclinometers and Horizontal Drains

As part of the previous work performed by Earth Tech Consultants, inclinometers and horizontal drains have been and are continuing to be installed along the slide. The inclinometers and horizontal drains are being monitored by the County of Santa Clara and Earth Tech Consultants.

5.0 SLOPE STABILITY ANALYSIS

5.1 General

In order to assess the failure mechanisms of the slope and to evaluate several repair alternatives, we performed a slope stability analysis using several different modeling methods and scenarios.

5.2 Geometry of the Slope and Failure Plane

To model the slope failure, the geometry of the failure surface had to be assumed based on the previous subsurface investigation and on our own field observations. Our geological investigation indicated that the bedding inclination of the bedrock underlying the slope dipped orthogonaly into the slope and therefore opposed the movement of the slide. Observations made in the field indicated that the slide is occurring in the highly weathered bedrock underlying the slope.

Based on the prior site investigation and on our field reconnaissance, we have assumed that the failure is a translational failure, which has occurred parallel to the joint and fracture plane of the underlying siltstone and sandstone bedrock. The failure zone, is indicated on cross-section shown on Plate 1B.

5.3 Analytical Tools

A stability analysis was performed using standard software models to develop a representative factor of safety of the slope, and to model our proposed slope repair scenarios. The software used in our analysis is described below:

5.3.1 XSTABL

XSTABL is a program that is used to perform two-dimensional, limit equilibrium analyses to compute the factor of safety for slopes. The program can be used to search for the most critical surface or the factor of safety may be calculated for a specified surface. For this analysis an irregular surface search was implemented using Janbu's simplified procedure.

5.3.2 FLAC

Fast Lagrangian Analysis of Continua (FLAC) is a two-dimensional finite difference program for engineering mechanics computation. FLAC is widely used in geotechnical applications, especially mechanical loading capacity and deformations analysis of slope stability. FLAC is suited to analyze large strain problems involving materials such as soil and rock that exhibit non-linear and plastic behavior. We have used FLAC to model the existing conditions and evaluate alternate remedial measures.

5.4 Sensitivity Analysis Using XSTABL

An initial sensitivity analysis was carried out using XSTABL to analyze the response of the slope to varying cohesive strengths, and friction angles of the soil in the assume slide zone. Additionally the phreatic water surface was varied in elevation to analyze the response of the slope to varying degree of saturation. The results of the sensitivity analysis of the slope are shown in Table 3a below.

It is noted that the factor of safety is defined as the resisting forces divided by the driving forces. Therefore a factor of safety of 1.0 indicates the driving forces equal the resisting forces and the slope is stable. If a slope is slowly moving along a slide plane its factor of safety is slightly less than 1.0, say 0.95. Man made fill slopes are usually designed to have a factor of safety of 1.5.

Table 3a -Sensitivity Analysis of the Slope

Run #	ϕ (deg.)	C (psf)	Depth to Water Table (ft)	FS	Remarks
1	18	550	NW	1.3	Block Analysis with no water in slope.
2	18	550	NW	1.5	Circular search pattern, the failure surface does not fit the observed data from field. The circles are passing through the upper rocky material and getting strength from soil above the slide plane. Try block analysis to better fit observed data.
3	22	550	NW	1.5	Block Analysis with no water in slope. The higher phi angle increases the stability of the slope but may be too high for failure under saturated condition.
4	15	550	NW	1.2	Factor is to low, will fail with any water added.
5	18	650	NW	1.4	Little change with increased c.
6	18	450	NW	1.3	Little change with decreased c.
7	18	550	40	1.2	
8	18	550	30	1.1	Marginal Stability
9	18	550	20	1.0	Marginal Stability
10	18	550	10	0.9	Slope Failure
11	22	550	40	1.3	
12	22	550	30	1.2	
13	22	550	20	1.2	
14	22	550	10	1.1	With slope near full saturation there is still no failure. Because failure has occurred this phi angle is too high.

Notes:

1. NW = No groundwater
2. Depth to water table is vertical distance from slope surface
3. FS = factor of safety is resisting forces/driving forces
4. A factor of safety below 1.1 is considered a marginally stable slope.
5. ϕ = Internal angle of friction in degrees
6. C = Cohesion in pounds per square foot.

Prior to running the analysis shown above, preliminary runs were performed to indicate likely ranges for the cohesive strength and the angle of internal friction of the soil along the failure plane. The choice of a cohesive strength of 550 pounds per square foot was based on the results of the torvane tests performed on the clay seam at the toe of the slide. A range of 15 to 22 degrees was considered to be appropriate for the internal angle of friction for the soil along the slide zone. In the computer models, the soil strength parameters were held consistent while varying the elevation of the groundwater to observe the change in the factor of safety of the slope. Because the slope failed during the winter, it was assumed that the factor of safety for the slope would approach a value of 1.0 as the soil became saturated. A factor of safety for the slope that falls below 1.0 is the theoretical point at which a slope failure will occur.

From our analysis we confirmed that the choice of 550 pounds per square foot for the cohesive strength and 18-degrees for the internal angle of friction were appropriate parameters for the clay gouge material in this slope. A circular failure surface was modeled and is shown as run No. 2 (Table 3a). The circular surface did not fit the failure surface which was located using the inclinometer data gathered at the site. A block planer surface gave the best fit to the observed movements and was utilized for all other runs using XSTABL.

Plate 3a shows the XSTABL runs using 550 pounds per square foot cohesive strength and the 18-degree friction angle with variations of the water table. Note that the factor of safety is below 1.5 in all of the analyses. Using these parameters, we found that the slope was marginally stable with limited saturation, and became unstable when the water table was within 20-feet of the slope surface. After the initial movement of the slide mass, the slide has stabilized somewhat and the movement is more characteristic of a creeping failure. If the factor of safety were to be well below 1.0, it would suggest the slope was in a catastrophic failure, which is not consistent with the field observations.

Plate 3b is an analysis of the lower part of the slide complex between Alum Rock Avenue and Penitencia Creek. A stability analysis was carried out to evaluate the effect of water table on the stability of the lower portion of the slide. We have assumed the same properties as we had in run numbers 7 to 10 in Table 3a. Table 3b below summarize the results of the sensitivity study for the lower portion of the slide.

Table 3b -Sensitivity Analysis of the Slope

Run #	ϕ (deg.)	C (psf)	Depth to Water Table (ft)	FS	Remarks
1	18	550	10	1.3	Marginal Stability
2	18	550	20	1.3	Marginal Stability
3	18	550	30	1.4	Marginal Stability
4	18	550	40	1.5	Acceptable Factor of Safety

Based on our analysis, a factor of safety of 1.5 against slope failure is achieved when the ground water table is lowered to depths of 40-feet below the surface. We have assumed that the ground water table reaches the ground surface towards the toe of the slope adjacent to Penitencia Creek flows.

We also performed analysis of the stability of Alum Rock Avenue as a result of:

1. Lowering the ground water and/or
2. Constructing a buttress fill between Alum Rock Avenue and Penitencia Creek.

We studied the effects of water table on stability of this slope and the results are summarized in Table 3a and Plate 3a. Plate 3c shows the slope stability analysis for several buttress configurations. For all of these analysis cases we kept the ground water table at 25 feet below the surface. The buttress fill is proposed to be an engineered compacted fill with ¾ to 1 slope.

An analysis was performed on four buttress fill repair alternatives and the results of the analysis are shown on Plate 3c. Our analysis assumed a geogrid reinforced fill with a ¾:1 inclination located at the toe of the existing slope above Penitencia Creek. The four alternatives were extended to elevations 160, 180, 200, and 220 feet respectively. The fill slopes were modeled as equivalent external vertical stresses acting over the surface of the existing slope. Our analysis indicates that only marginal improvement in the factor of safety can be expected from the placement of a surcharge fill. Table 3c below summarizes the results of our analysis.

Table 3c –Buttress Fill Stability Analysis

Run #	Buttress Fill	Water Table Elevation	FS	Remarks
1	No Fill	25'	1.0	Marginal Stabilization No Buttress Fill
2	¾:1 Fill to 160 foot elev.	25'	1.08	Marginal Stabilization
3	¾:1 Fill to 180 foot elev.	25'	1.09	Marginal Stabilization
4	¾:1 Fill to 200 foot elev.	25'	1.10	Marginal Stabilization
5	¾:1 Fill to 220 foot elev.	25'	1.11	Marginal Stabilization

It is also noted that a buttress fill would be very difficult to construct. The excavation would require steep potentially unstable cuts and a large stockpile area. The size of the excavation would be enlarged beyond the slide area to accommodate ramped access roads. Therefore, the feasibility of a buttress fill is questionable at best.

5.5 FLAC Parametric Analysis

5.5.1 Model Setup and Material Parameters

A FLAC analysis was performed to model the existing and proposed conditions at the site. The analysis was carried out to model the deformations at the site using the surface topography and subsurface conditions at the site.

The rationale behind the material parameters selected in the FLAC analysis is the same as that described in the limit equilibrium approach. Material parameters used in the analysis are summarized in the table below.

The material properties were used in analyzing the existing conditions of the slide as to model proposed remediations. The model was set up using the cross-section represented by Section B-B on Plate 1B. The surface and sub-surface data were input along with the observed slide plane location used in the previous models.

Table 4: Soil Properties

No.	Discription	Density (pcf)	Cohesion (psf)	Friction Angle (deg)	Bulk Modulus (psf)	Shear Modulus (psf)	Poissins Ratio
1	Shale/Siltstone/Sandstone-weathered & fractured	130	1000	32	2.00E+06	2.00E+05	0.30
2	Clay Gouge/Slide plane	120	550	18	1.00E+06	1.00E+05	0.40
3	Shale Siltstone Sandstone-Intact Rock	140	2000	45	3.80E+08	2.10E+08	0.25

Plate 4 shows the finite difference grid, geometry and the boundary conditions used in the model set up. The mesh was set up to vary the grid size with more data points along the slide plane. The horizontal and vertical boundaries of the model were extended far enough away so that the boundary conditions would not interfere with model response. Once the model grid, boundary conditions, and material parameters with no groundwater were established, gravity load was applied to the model and all displacements and unbalanced forces were allowed to reach equilibrium. The static horizontal and vertical stress distributions were plotted and are shown in Plate 5. These values of stress distributions were checked by simple geostatic hand calculation, which confirmed model accuracy.

5.5.2 Introduction of Groundwater Model

Once the "no" groundwater conditions were established and confirmed, water was included in the model. Based on the groundwater observations from the wells and the sensitivity and Limit equilibrium analysis, the water table was varied from 10 to 40 feet below ground surface. Table 5 provides a summary of all the analysis cases that we ran in studying the effect of groundwater to slope stability. Four groundwater level conditions were analyzed for 40, 30, 20, and 10 feet below ground surface. The results confirm the slope becomes unstable, i.e. large displacements begin to occur, once the water level rises from 30 to 20 feet below ground surface. Plates 6 and 7 show the results for the 30-foot condition. Plates 8 to 9 show the results for the groundwater being at 20 feet below ground surface. The values of displacement shown in the table represent a total displacement as a result of groundwater being maintained at the respective level for the entire duration of the model run.

We understand that the horizontal drains have been installed in the upper portion of the slope. As pointed out in the report by Earthtech Consultants, the horizontal drains are to be considered at best a temporary fix to the stability of the slope. Due to clogging and uncertainties regarding future rainfall they can not provide a high degree of confidence of the slope's stability. We do not recommend relying on the horizontal drains to maintain the long term stability of the slope. For the remainder of the analysis, groundwater was held to 20 feet below ground surface.

Analysis Case	Description	Depth to Ground Water (ft)	Disp. @ Alum Rock Rd.(ft.)	x-disp ft.	y-disp ft.	Remarks
1	Existing slope	40.0	0.12	0.08	0.08	Slope is Stable
2	Existing slope	30.0	0.39	0.30	0.25	Slope becomes barely stable after small displacement
3	Existing slope	20.0	17.50	15.00	10.00	Becomes stable after unacceptable levels of displacments
4	Existing slope	10.0	21.70	20.00	12.50	Slope fails
5	Vert. 3 ft. diam piers @ 9ft. o.c.with tiebacks @ 1 ft. o.c.	20.0	12.00	10.00	6.00	Excessive displacement
6	Vert.4ft. Diam piers @ 8 ft. o.c. with tiebacks @ 1 ft. o.c.	20.0	3.00	2.00	1.00	Excessive displacement
7	A-frame piers @ 5 degrees with tiebacks @ 4 ft. o.c.	20.0	0.90	0.75	0.25	Acceptable displacement
8	A-frame piers @ 20 degrees with tiebacks @ 4 ft. o.c.	20.0	0.40	0.25	0.10	Acceptable displacement

5.5.3 Structural Slope Stabilization

We analyzed a pier and grade beam retaining structure with tie backs as a method to control further slide movement. Several pier and tied back configurations were evaluated. The configurations and the resulting displacements are summarized in Table 5 above.

The analysis shows that a vertical pile with very frequent tie-back spacing does not reduce the slope movements to acceptable levels.

Vertical Drilled Piers

Plates 10 to 13 shows a vertical pier and grade beam with tie-back analysis case. The piers were 4-foot diameter piers with a spacing of 8 feet on center horizontal spacing. When the groundwater is 20 feet below ground surface, the slope becomes unstable and the 55-foot thick slide mass begins to impart lateral and compressive loads to the grade beam and tension on the tie backs. The magnitude of displacements at the end of the run is shown in Table 5 and was excessive to the point of failure.

Inclined/Battered Drilled Piers

A 5-degree inclination to the drilled pier greatly enhances the performance of the structural system. Plates 14 to 17 shows the lateral shear stress contours, displacement vectors, displacements, and x and y displacement contours for the case of 5-degree battered drilled piers and grade beam with the tied back system. Plates 18 to 21 show similar plots for 20-degree inclined drilled pier system. Piers inclined at 20 degrees showed significant improvement in performance. There is a direct relationship between pier inclination and slope displacements/stability. However, from a practical stand point we suggest a 5-degree inclination of the piers because it may not be possible to install structural steel into a pier with a 20-degree batter.

6.0 REPAIR ALTERNATIVES

6.1 General

We analyzed three alternatives for increasing the stability of the slope. The three alternatives include (1.) Lowering the ground water using horizontal drains and a deep drainage gallery near the top of the landslide, (2.) replacing the upper slide mass with a reinforced earth fill, and (3.)

constructing an A-Frame tied-back drilled pier wall located at mid-slope below Alum Rock Avenue.

We understand that the County installed horizontal drains in the upper half of the slope as an interim mitigation measure in accordance with Earth Tech's proposal. We performed a stability analysis assuming that the horizontal drains will hypothetically, be effective in lowering the groundwater to a depth of thirty feet below the ground surface assuming that additional drains would be installed in the lower half of the slide. Based on this assumption, the results of our stability analysis indicate that the factor of safety against further slope movement will be marginal at 1.09. In our opinion, lowering the groundwater to this level, with no other repair scenario utilized, will not be sufficient to stabilize the slide movement. It has not been established that the drains are sufficient to lower the water table to this level during the rainy season. If a deep drainage gallery were installed near the top of the slide on Highland Drive, further dewatering to 40 feet or more may be possible. Our analyses indicate that a water level 40 to 50 feet below the ground surface would provide a marginal factor of safety of 1.2

6.2 Deep Drainage Gallery

The deep drainage gallery would consist of 3-foot diameter shafts to depths well below the slide plane of 100 to 150 feet spaced at 20 feet along Highland Drive for a distance of about 260 feet. These shafts would be hydraulically connected at the bottom by a micro-tunneled drain installed outside one of the limits of the landslide and discharging at the bottom of the slope. The shafts would be back-filled with Class 2 Permeable Material to within 10 feet of the surface and capped with less permeable clayey backfill and pavement. Schematic views of this system are shown on Plate 24. Installation of the micro-tunnel drain will require very sophisticated trenchless technology to accurately locate the drain within the tolerances of the shafts and their predetermined locations. The deep drainage gallery would be compromised by continued slope movement that could shear off the three foot diameter shafts. For this reason the deep drainage gallery is not the most desirable solution.

6.3 Mass Grading – Reinforced Earth Slope

In order to reduce the driving force of the slide and repair Crothers, Alum Rock Avenue, and the Maintenance Roads, we analyzed a slope removal and replacement alternative. This alternative involves excavating the upper slide mass to a level extending below the slide plane (a depth of approximately 50-feet). Blanket drains would be installed and a new reinforced earth fill embankment constructed. The finite difference analysis indicates that after the installation of the fill, the lower slide mass would move approximately 2-feet and then come to equilibrium. The upper reinforced earth fill would be stable. The difficulty with this alternative would be construction access. Constructing a fifty-foot deep excavation on top of the slope would create a large quantity of soil from the slide debris which would have to be temporarily stockpiled. Care would have to be taken so as not to aggravate another slide by the surcharging of the stockpile. As a practical matter, this alternative's feasibility is questionable. This alternative is shown on Plate 22.

6.4 A-Frame with Tie-Backs

The third alternative we evaluated consists of an A-frame, tied-back, drilled pier retaining structure as shown in Plate 23. The retaining structure would be located below Alum Rock Avenue, so as to stabilize the uppermost and steepest part of the slide area. We estimate that the drilled piers would be 90 to 110-feet deep and 4-feet in diameter. The piers would be alternately splayed approximately 5 degrees uphill and downhill. The piers would be secured by an approximately 10-foot wide grade beam at the top. Lateral resistance would be increased with tie-rods extending approximately 150 feet into the hillside, or about 100 feet beyond the slide plane. The 4-foot diameter drilled piers can be spaced 8 feet on center. The tie rods can be spaced at 4 feet on center and should be connected to the grade beam. This alternative can be constructed with minimal disruption to the existing hillside. In the short term, the drilled piers would act as shear pins and reduce slope movement immediately after they were installed. In the long term, the tied-back retaining structure would further reduce slope movement in the upper 50-feet of the hillside above Alum Rock Avenue by increasing the factor of safety to 1.2 or greater. We estimate the lower slide mass below Alum Rock Avenue would continue to creep but would stabilize after slope deformations of about 4-feet.

The slope replacement and the retaining structure alternatives both assume that the horizontal drains that were installed would be effective in reducing the groundwater level to approximately 20-feet below grade.

The structural design of the A-frame will require very large bending and load capacities in the members as well as the tiebacks. Our limit equilibrium slope analyses indicates that with the current ground water levels of about 25 feet below ground surface, a resultant force on the frame of 144 kips per foot would be necessary to achieve a factor of safety of 1.2 for the slope. To get another handle on the magnitude of these loads, we performed a FLAC analysis for the A-frame reducing the shear strength of the slide plane by a 1.2 factor of safety. We can provide full details of the displacements, shear and bending stresses at the design phase of this project if a decision is made to go forward.

7.0 COST ESTIMATE AND SCHEDULE

7.1 Cost Estimate

We have prepared the following cost estimate for the Tie-back wall and roadway improvements.

Tie-back Wall

Feasibility and preliminary Engineering -	$100,000
Final Design	$200,000
Construction	$1,400,000
Construction Management & Contingency	$300,000
TOTAL	$2,000,000

Roadway Improvements

Design	$100,000
Construction	$1,000,000
Construction Management & Contingency	$300,000
TOTAL	$1,400,000

7.2 Schedule

Tie Back Retaining Structure Design	Year 1
Construction	Year 2
Road Restored	Year 3

8.0 LIMITATIONS

The opinions and recommendations presented within this report are based on the visual observation of the site and the subsurface data contained in reports performed previously by others. Additional laboratory testing and analyses will be performed to develop design level geotechnical recommendations. The services provided under this contract as described in this letter report include professional opinions and conclusions based on the data collected and reviewed. These services have been performed according to generally accepted geotechnical engineering practices that exist in the project area at the time this letter report was written. This report is for project scope analyses purposes only and is not intended to serve as a design level geotechnical engineering report. No warranty is expressed or implied.

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APPLICATION FOR AUTHORIZATION TO USE
GEOTECHNICAL ANALYSIS REPORT
FOR LANDSLIDE STABILIZATION
ALUM ROCK PARK ENTRANCE, SAN JOSE, CALIFORNIA

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